Psuedo-static analysis of piles in liquefied soils: a case study of a bridge foundation

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ABSTRACT

In this paper, a pseudo-static analysis of piles in liquefying soils is applied to a case study of a bridge foundation. The response of piles is separately evaluated for the cyclic phase during the intense shaking and development of liquefaction, and for the subsequent lateral spreading phase. Effects of key parameters influencing the pile response are examined through parametric analyses with a particular attention being given to the variation in stiffness and residual strength of the liquefied soil. The results shed light on the relative importance of key parameters for different combination of loads and ground conditions, and allow comparative evaluation between loads on the pile exerted by the crust layer and the liquefied layer.

1 INTRODUCTION

Soil liquefaction has caused major damage to pile foundations in previous earthquakes, particularly the 1964 Niigata and 1995 Kobe events. Many methods are available to analyse the seismic response of pile foundations, including pseudo-static analysis, a simple design orientated approach. This analysis can be performed using common site investigation data such as SPT blow count, yet it captures the basic mechanism of pile behaviour.

The phenomenon of soil liquefaction and lateral spreading is complex and predictions of the seismic response are subject to a high level of aleatoric uncertainty. This suggests that when simplified analysis is performed, the key consideration is not the modelling itself; rather it is dealing with the uncertainties in a sensible manner. This paper describes analysis of piles in liquefied soil using a pseudo-static approach where the key input parameters are varied parametrically to identify key features of the response. In the analyses, the cyclic phase of the loading and subsequent lateral spreading phase were considered separately since the loads and soil conditions are greatly different between these two phases.

2 ANALYTICAL MODEL

The analytical model used in this paper is based on simplified three layer model described in Cubrinovski and Ishihara (2004), that consists of a crust layer, liquefied layers and a non-liquefied base layer. Bi-linear p-δ relationships for the soil layers and a tri-linear M-φ relationship for the pile are used for modelling the nonlinear behaviour of the soil and the pile respectively, as shown in Figure 1. In the analysis, lateral ground displacements representing either cyclic ground displacements or lateral spreading displacements of liquefied soils are applied to the pile. Note that these are free field ground displacements unaffected by the pile foundation.

Input parameters of the model are summarized in Figure 1 where k is the subgrade reaction coefficient, p is the ultimate soil pressure, U_{G2} is the lateral displacement at the ground surface and β is stiffness degradation factor for the liquefied soil. A discrete FE beam-spring model for the soil-pile system, as shown in Figure 1b, allows definition of different values for the stiffness and ultimate pressure of the soil and hence provides more rigorous and versatile analysis. Such beam-spring model was adopted in this study.

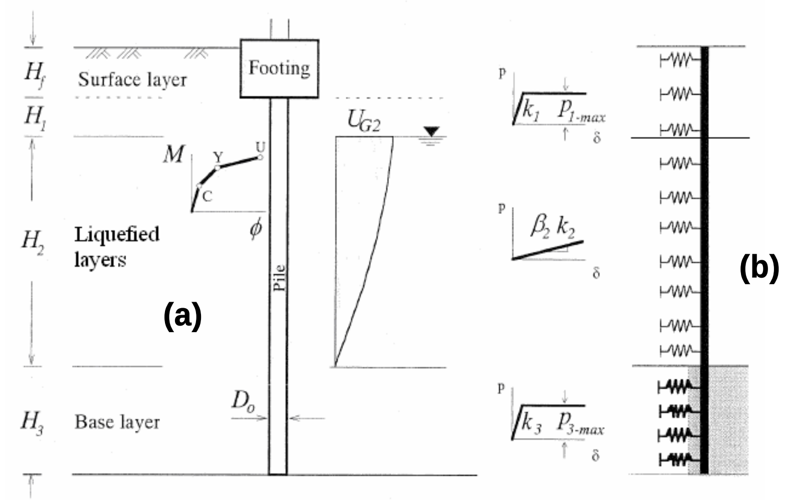

Figure 1. Analytical model for psuedo-static analysis: (a) characterisation of soil and pile non-linear behaviour; (b) discrete FE beam spring elements

3 CASE STUDY

To illustrate the application of the procedure described above a case study of a bridge founded on pile foundations in liquefiable soils is presented. A parametric study is also performed to demonstrate the effects of key parameters on the pile performance.

3.1 Description and ground profile

The case study is of twin bridges crossing the Avon River in Christchurch, New Zealand. The bridge has been identified as an important lifeline, and a structural retrofit has been proposed by the City Council to reduce the risk of failure in an anticipated Alpine fault event. Site investigations reveal that the stratigraphy and characteristics of the site vary significantly. The analysis presented in this paper is for a single pile located at the position of the site with the poorest ground conditions and hence represents a conservative assessment. The effects of the stiffer soils, pile group effects and soil-structure interaction were also considered but are not discussed herein. The proposed 1.2m diameter reinforced concrete pile extends from 2.5m to 22.5m below ground level. The pile is rigidly connected to a 2.5m deep abutment. Figure 2 shows the results of the site investigation and the assumed soil profile and SPT blow count for each layer. The water table depth varies from 2 – 2.5m, and the soil between 2.5m and 17.5m is considered to be liquefiable layers of sandy gravel and silty sand, with a dense silty sand base layer below 17.5m depth.

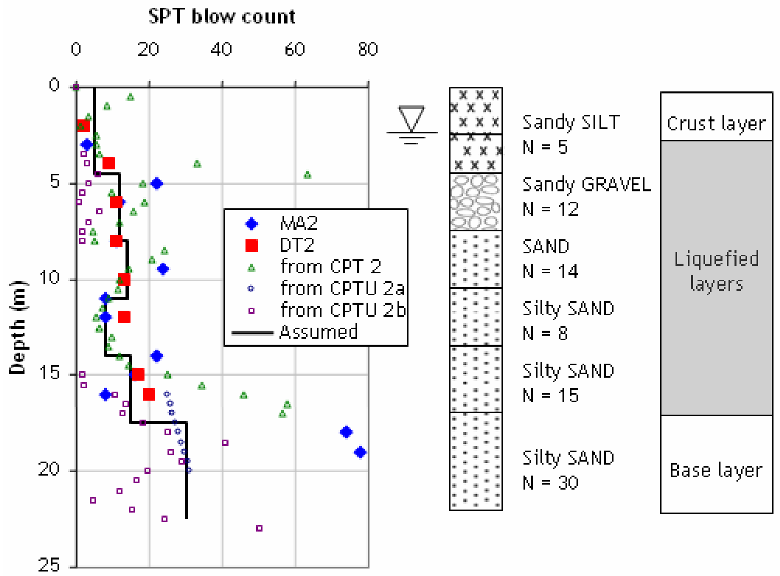

Figure 2. Soil profile and SPT blow count used in modelling

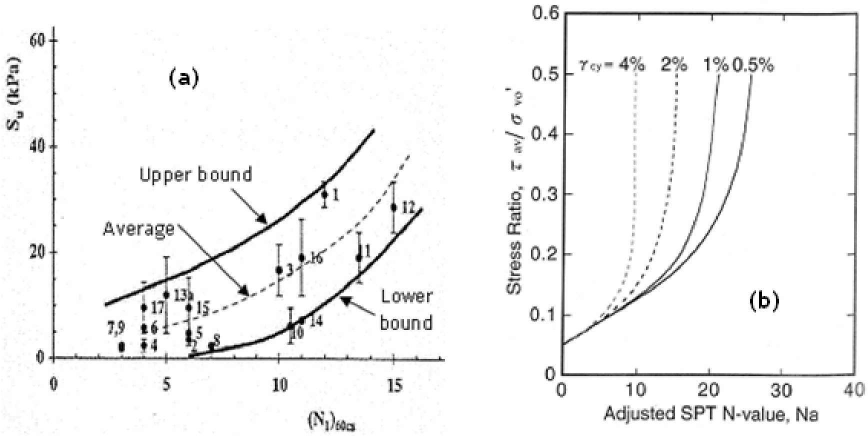

Figure 3. Empirical charts used to evaluate: (a) ultimate pressure exerted by liquefied soil through undrained shear strength (Seed and Harder 1991); (b) cyclic ground displacements through induced shear strains (Tokimatsu and Asaka 1998)

3.2 Determination of model parameters

Cyclic ground displacements in liquefied soils were estimated using the simplified procedure described in Tokimatsu and Asaka (1998). The procedure is based on observations from previous earthquakes, where the cyclic shear strain was evaluated from analysis of strong motion records and detailed surveys of piles in level ground and then plotted against SPT value as shown in Figure 3(a). The chart is essentially equivalent to the conventional SPT-based charts for evaluation of liquefaction. For each liquefied soil layer the cyclic shear strain was first evaluated and then the cyclic ground displacement profile was calculated by integrating the shear strains throughout the soil profile. The cyclic ground displacement at the surface of the investigated profile was predicted to be 40 cm.

The subgrade reaction coefficient, κ, for the soil layers were calculated using the following empirical formula $\kappa = 56\ N\ D_o^{-3/4}$ [MN/m^3] where N is the SPT blow count and D_o is the pile diameter in cm. For both crust layer and base layer, the ultimate pressure, p_{max}, exerted by the soil on the pile is calculated as the Rankine passive pressure multiplied by a factor α_u. This factor is introduced to account for the difference in lateral pressure between a single pile and an equivalent wall. α_u was assigned a value of 4.5 for piles based on the results of a full-size test (Cubrinovski et al. 2006), while a value of 2.0 was adopted for the abutment, due to consideration of pile group effects.

The interaction in the liquefied layer can be treated in a simplified manner by an equivalent linear p-δ relationship, i.e. with no ultimate pressure. Alternatively and more rigorously, a limit can be placed on the pressure exerted by the liquefied soil. One approach in doing this is to use the undrained or residual strength, S_u, defined by Seed and Harder (1991) from empirical correlations with SPT value, as shown in Figure 3(b). Since the scatter of the data is quite significant, three S_u values were considered in this study corresponding to an upper (S_{u-ub}), average (S_{u-ave}) or lower bound value (S_{u-lb}), as indicted in Figure 3(b).

3.3 Parametric study

Due to the level of uncertainty in predicting: (a) the stiffness of the liquefied soil, (b) the likely inertial load from the superstructure, (c) the ultimate pressure from the liquefied soil and (d) the likely ground displacement, a parametric study was conducted to observe how these parameters affect the pile response.

In the cyclic shaking phase analyses, the β value was varied between 1/10 and 1/50. The inertial load was equal to the axial load on the pile multiplied by the peak ground acceleration (0.44g). For purpose of comparison, analyses with no inertial load were also

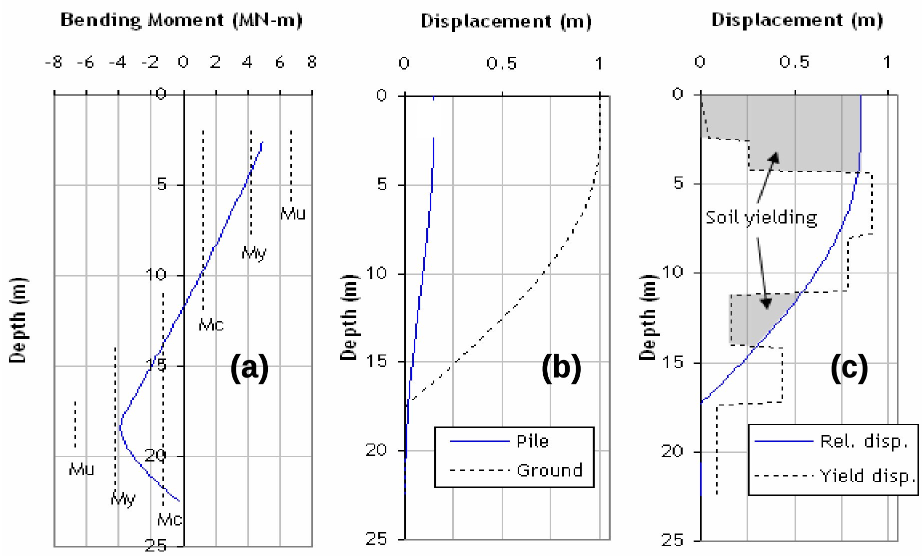

Figure 4. Typical analysis result showing the pile response to lateral spreading of one metre: (a) bending moment versus depth plot; (b) pile and ground displacements; and (c) relative displacement between the soil and pile compared to the soil yield displacement

performed for all analysis cases. The analysis of the lateral spreading phase was conducted with β varying between 1/50 and 1/1000, and the ground displacement was either 1 or 2m with a cosine distribution throughout the liquefied layer. All lateral spreading analysis cases were conducted without inertial load. Effects of limiting the ultimate pressure from the liquefied layer were examined by comparing results of analyses using an equivalent linear or bi-linear p-δ relationships for the liquefied soil. In the latter case, the value of the ultimate pressure for the liquefied layer was varied between the average, upper and lower bound values indicated in Figure 3b.

3.4 Typical results

The results from one case are described in detail to show features of the response. Figure 4 shows the results for analysis case L13, where the lateral displacement was 1m, and in the liquefied layer the β = 1/1000 and $S_{u\text{-}lb}$ was used to limit the ultimate lateral pressure. Figure 4(a) shows the pile bending moment distribution with depth, with reference to the cracking, yield and ultimate moments. It can be seen that the maximum moments occur at the pile head and at the interface between the liquefied and base layers. Figure 4(b) shows the pile displacement compared to the ground displacement, exhibiting stiff pile behaviour resisting the movement of the ground around it. Figure 4(c) shows the relative displacement between the soil and the pile plotted with the soil yield displacement. This indicates the parts of the soil profile where the soil is yielding, i.e. where the limit on the ultimate pressure has been reached.

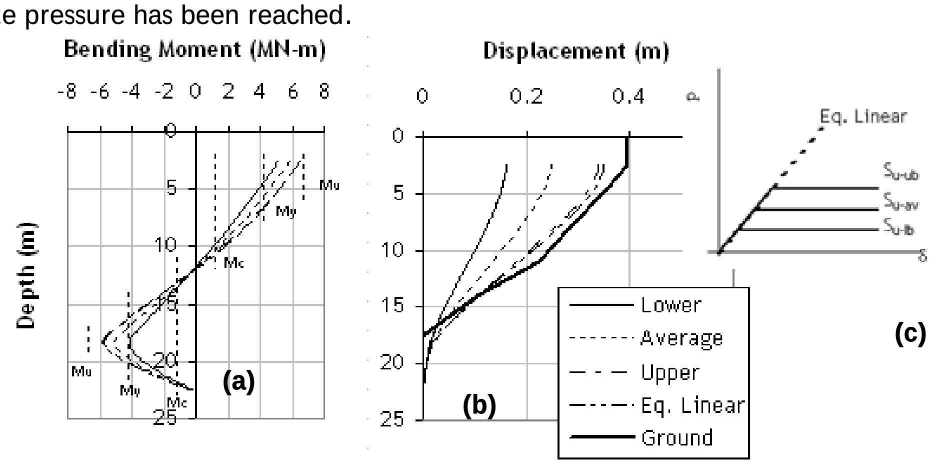

Figure 5. Pile response for β = 1/10 and no inertial load, showing the effects of changing the ultimate liquefied pressure.

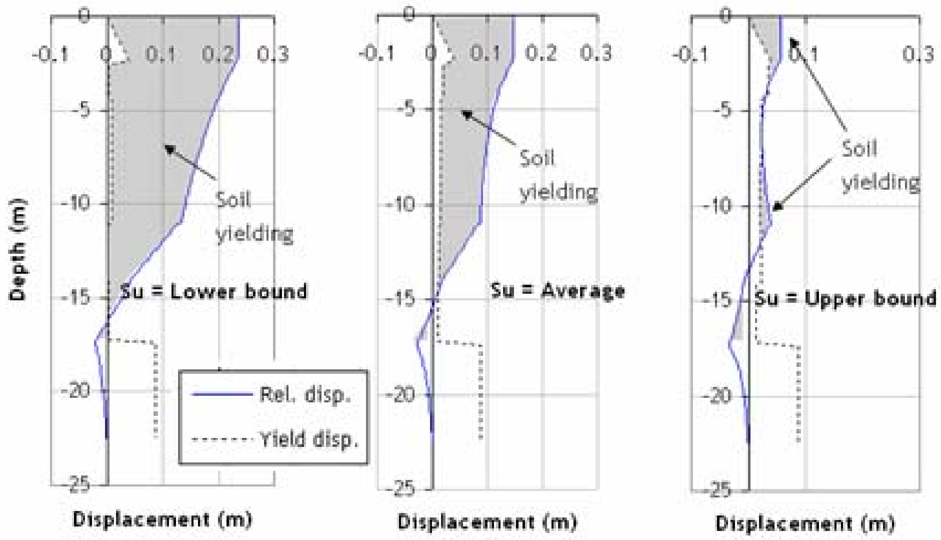

Figure 6. Comparison of relative and yield displacements for different values of S_u

4 RESULTS AND DISCUSSION

4.1 Overview

As expected, the largest pile response for the cyclic phase was obtained when a large inertial load was applied. That loading combination caused the top 5m of the pile to yield, as well as a 4m length at the interface between the liquefied and base layers. The worst case lateral spreading loading caused the moment to approach the failure moment of the pile at the pile head and also caused yielding at the interface as above.

4.2 Effects of inertial load and crust layer depth

When no inertial load is present the pile exhibits stiffer behaviour. The peak bending moment and the pile head displacement were reduced by 20-30% and 30-50% respectively. The effects of varying the crust layer depth from 2.0 to 2.5m to account for fluctuating water table depth were relatively small on the overall pile response.

4.3 Effects of liquefied layer properties

The results of cyclic shaking phase analyses show that with a large inertial load applied the results are not sensitive to changes in the β value. Furthermore, the selection of a limit on ultimate pressure from the liquefied layer had no effect on the response. The large inertial load results in flexible pile behaviour, i.e. the pile moves with the soil. Hence there are small relative displacements, which results in small loads from the liquefied layer and the lateral pressure limits never being reached.

The effects of changing β values and limiting pressures become apparent without an inertial load applied, as shown in Figure 5. The upper bound case (using S_u-maximum) is very similar to the equivalent linear case, but the average and lower bound values for S_u show decreased bending moments and pile displacements. These observations can be explained by considering the relative displacements for the three cases with limiting pressures. Figure 6 shows the relative displacement between the soil and the pile plotted with the soil yield displacement for the three cases of S_u value. It can be seen that the lower bound S_u case has low yield displacements and high relative displacements, whereas the upper bound S_u case has higher yield displacements and lower relative displacements. A large part of the soil profile has yielded with the lower bound case, so despite having larger relative displacements than the upper bound case the pressure acting on the pile is much lower. Figure 6 also explains why the upper bound case is so similar to the equivalent linear case, as only a small portion of the soil profile is yielding.

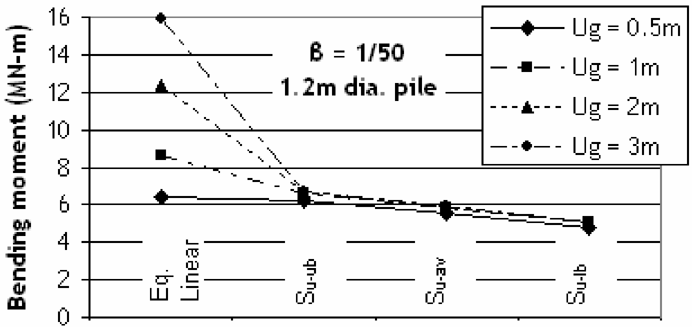

Figure 7. Variation of bending moment with lateral spreading distance for different values of ultimate pressure from liquefied soil

The relative contributions of the crust and liquefied layers also change with the value of S_u. As the value of S_u decreases the role of the liquefied layer diminishes; for the case shown in Figures 5 and 6 the contribution of the liquefied layer to the total load decreased from 50% in the equivalent linear case to 27% in the $S_{u\text{-}lb}$ case. Also, as the crust layer remains unchanged, total load on the pile decreases, resulting in the behaviour shown in Figure 5.

Changing the value of β used in the analyses caused a large difference in the pile response, when $\beta = 1/50$ the peak bending moments and pile head displacements are much larger than the $\beta = 1/1000$ case. Variation in the ultimate pressure exerted by the liquefied soil also had a large effect on the pile response. The reasoning behind placing limits upon the ultimate pressure exerted from liquefied soil is to avoid unrealistic loads being imposed in situations with very large ground displacements. Figure 7 shows how the bending moment of a 1.2m diameter pile with a β value of 1/50 varies for different levels of ground displacement. It can be seen that when limits are placed on the ultimate pressure the different levels of ground displacement yield virtually the same response. This is hardly surprising given that in these cases the vast majority of the soil profile is yielding, thus exerting the same pressure on the pile.

5 CONCLUSIONS

A case study of the psuedo-static analysis of a pile in liquefiable soil has been presented. To understand the effects the selection of input parameters have on the response of the pile a parametric study has been conducted. Key findings include:

- For both the cyclic shaking and lateral spreading phases, results of the simplified analysis indicate that the pile will yield near the pile head and at the interface between the liquefied and base layers when the largest loads are applied.
- The effects of changing the stiffness of the liquefied soil were not significant when a large inertial load was applied
- Reducing the ultimate pressure results in a decrease in the pile response and a decrease in the contribution of the liquefied layer to the overall response
- When ultimate pressure is used for the liquefied soil layers the pile response is not affected by the magnitude of ground displacement once a certain ground displacement has been reached

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